DESIGN RECOMMENDATIONS FOR STRONGBACK RETROFITS

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ABSTRACT

Since the early 2000s, it has been widely recognised that existing hollow-core floors can pose a threat to life during earthquakes, but only limited and mostly unvalidated guidance on how to retrofit the many existing hollow-core floors in New Zealand has since been provided. The lack of retrofit validation was emphasised by recent experimental findings showing that seating angles do not necessarily address positive moment failure within the precast floor unit, as has commonly been assumed.

A new retrofit solution referred to as 'strongback' retrofit has been developed and experimentally validated. The strongback retrofit consists of short steel beams running longitudinally underneath the hollow-core unit combined with a supplementary support. When the floor loses its gravity load-carrying capacity, the strongback retrofit can provide a robust alternative load path. Thereby, the strongback retrofit can address the majority of known hollow-core floor failure modes, including positive moment failure proximate to the support.

This paper provides design recommendations for the new strongback retrofit and describes the key performance observations from the validation test. The paper also discusses in what situations the strongback retrofit should be used.

1 BACKGROUND

Precast hollow-core floors were a widely used flooring solution in New Zealand, particularly during the 1980s to 2000s. However, the 1994 Northridge Earthquake and the 2016 Kaikoura earthquake (Henry et al. 2017) exposed critical vulnerabilities of these floors under seismic demands. Guidance for assessing the drift capacity of various failure modes is provided by Fenwick et al. (2010) and the Technical proposal to revise Section C5 (Concrete Buildings) of the "Guidelines for Detailed Seismic Assessment of Buildings" (MBIE et al. 2018). The assessment guidelines distinguish between three primary failure modes, namely loss of support (LOS), negative moment failure (NMF) and positive moment failure (PMF).

While there is well-considered guidance for the assessment of hollow-core floors available, only limited guidance has been provided on retrofit solutions for the many existing poorly detailed hollow-core floors in New Zealand. To date, the most commonly used retrofit has been supplementary seating retrofits placed under the soffit of the floor unit and bolted to the face of the supporting beam. The primary intent of this retrofit has been to address failure due to LOS, but it has also been widely assumed to address PMF. However, the success of a supplemental seating retrofit in addressing PMF

depends on two conditions. Firstly, the positive moment crack needs to form close to the beam face, and secondly, the crack needs to be steep so that the broken end of the floor still has sufficient shear capacity to transfer the gravity loads into the seating angle. The PMFs observed in early hollow-core floor tests (Bull and Matthews 2003; Matthews 2003) fulfilled these two conditions, as shown in Figure 1. As a result, it was commonly assumed that supplemental seating retrofits can address PMF.

Figure: 1 Steep positive moment crack along the ledge of the seating resulted in floor collapse (Bull and Matthews 2003)

(a) Transverse soffit cracking away from the support and seating angle (Henry et al. 2017)

(b) Internal cracking at a shallow angle (Büker et al. 2021)

Figure 2: Cracking in hollow-core floors

Despite the early indications that supplemental seating retrofits could mitigate the risk of collapse due to PMF, evidence from the 2016 Kaikoura earthquake (Henry et al. 2017) demonstrated that positive moment soffit cracks can form up to ~400 mm away from the support (e.g. Figure 2a), making the typical supplemental seating retrofit ineffective if damage progresses at this location. In addition, recent testing (Büker et al. 2021, 2022b) showed that the slope of the web cracks that extend from the positive moment soffit crack could, in fact, be shallower (approx. 30 degrees, as shown in Figure 2b) than previously observed. In such cases, even if the positive moment crack forms close to the beam face, the remaining shallow concrete wedge seated on the angle cannot reliably support the weight of the floor (Büker et al. 2021). Currently, neither the slope of the web crack nor the location of the transverse soffit crack can be predicted reliably. As a result, seating angles need to be viewed as insufficient for PMF (SESOC et al. 2021).

A new retrofit solution has been developed that can address the uncertainty of both the location of the positive moment crack and the slope of the associated web crack. This new retrofit is referred to as a 'strongback' retrofit and can, in fact, address the majority of hollow-core floor failure modes. The effectiveness of the strongback retrofit was successfully demonstrated in a super-assembly test recently conducted at the University of Canterbury (Büker et al. 2021, 2022b).

Design and detailing recommendations for the strongback retrofit have been developed under consideration of the performance observation from the experiment. These recommendations are described in the subsequent sections of this paper.
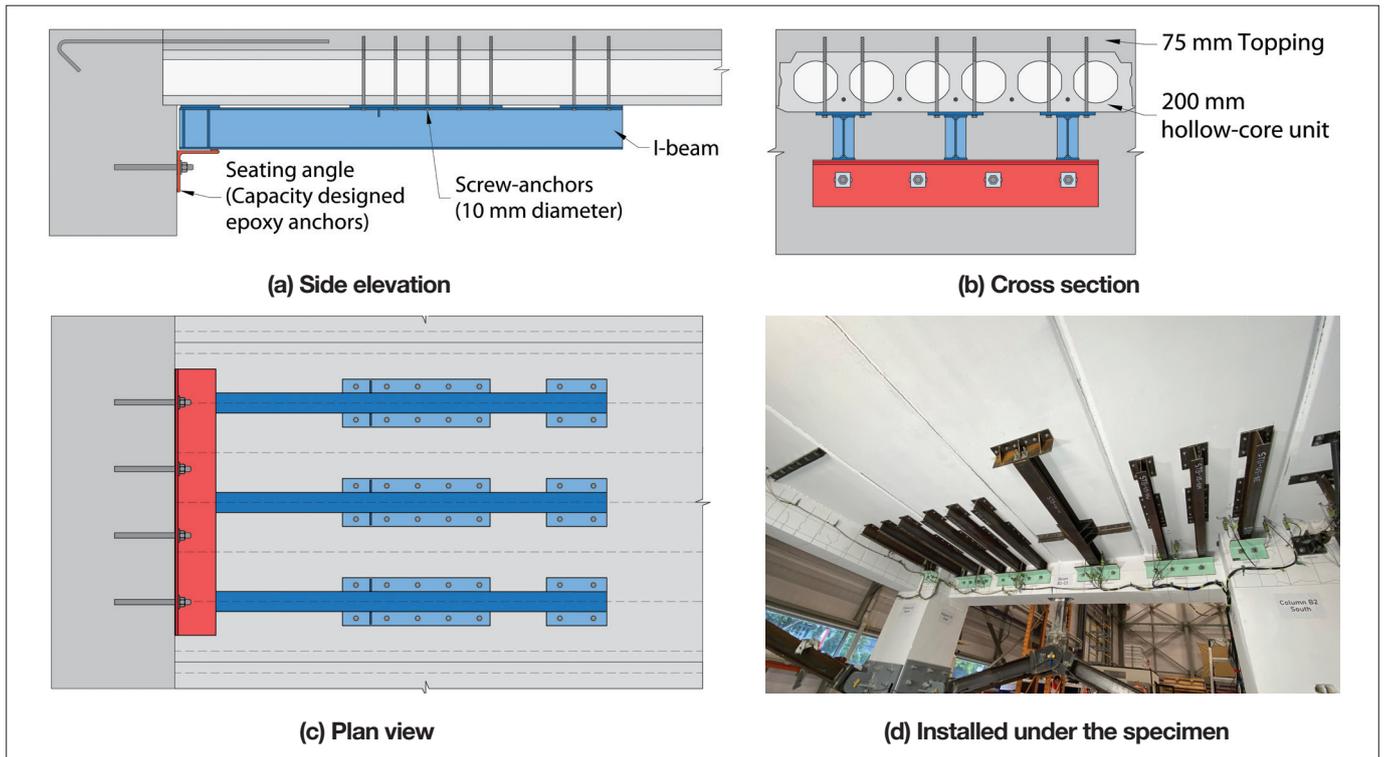
2 STRONGBACK RETROFIT CONCEPT

The strongback retrofit is a versatile retrofit solution that addresses most hollow-core floor failure modes. It is notably effective for flooring units prone to PMF and NMF. The strongback technique can also be used to reinstate a reliable load path where existing damage, such as web or soffit cracking, compromises the capacity of the end region of a hollow-core unit.

Experimental validation of the strongback retrofit was conducted as part of a super-assembly test (Büker et al. 2021, 2022b), which is subsequently referred to as the 'validation test'. The performance of the strongback retrofit observed during the validation test is described at various relevant points in this paper. The drawings in this section generally reflect the retrofit detailing used in the validation test, but modifications to improve the detailing are incorporated in the drawings shown in the following sections.

The main components of the strongback retrofit, illustrated in Figure 3, are steel members running longitudinally underneath the hollow-core unit. These steel members are seated on a seating angle bolted to the support structure and, at the opposite end, fixed to the floor by post-installed anchors. Another point of contact is provided by a steel plate with screw anchors located at the desired position of a compression reaction.

The conceptual idea of the strongback retrofit is to provide an alternative load path if the floor loses its gravity load capacity. During the formation of an NMF in the floor unit, for instance, the shear capacity of the end of the hollow-core unit diminishes in a sudden, brittle fashion. The strongback retrofit prevents floor collapse by transferring the loads from the intact part of the floor beyond the failed section to the supporting beam via the seating angle.
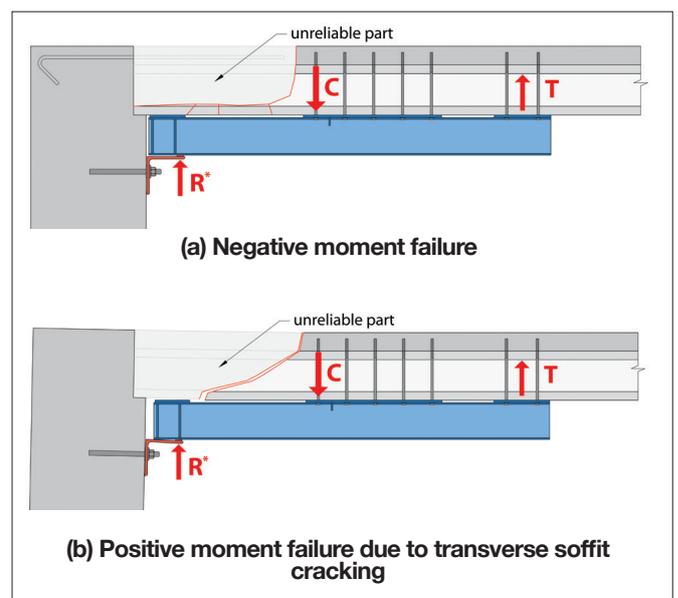
(a) Side elevation

Seating angle
(Capacity designed
epoxy anchors)

Screw-anchors
(10 mm diameter)

I-beam

(b) Cross section

75 mm Topping

200 mm
hollow-core unit

(c) Plan view

(d) Installed under the specimen

Figure 3: Strongback retrofit as used in the super-assembly test

This mechanism is demonstrated in Figure 4a, where the structurally unreliable part of the floor is shown as faded. Also indicated in this figure is the compression-tension force couple, which is required to resist the moment and shears generated by the transfer of forces from the hollow-core unit to the strongback. Similarly, this alternative load path can be utilised when a PMF occurs, as illustrated in Figure 4b.

In the validation test, the strongback retrofit was also installed under the hollow-core unit that spanned right next to the adjacent beam, which is commonly referred to as the 'alpha unit' (Brooke et al. 2022). An alpha unit undergoes incompatibility displacement demands with the adjoining beam deforming in double curvature while the alpha unit tries to remain straight. The relative displacement demands between the beam and the floor can lead to horizontal splitting of the hollow-core floor webs. When the web-splitting develops and propagates further under increased inter-storey drift demands, the bottom flange of the hollow-core unit may collapse (Matthews 2003). The strongback configuration tested in the validation test was able to restrain the web-splitting due to the presence of screw anchors. Despite the successful prevention of this failure mechanism in the validation test, it is important to recognise that the tested hollow-core units only spanned one bay. For an alpha unit spanning one bay, the largest deformation due to displacement incompatibility occurred towards

the ends of the hollow-core unit, where the strongbacks with the screw-anchors were located. In cases where alpha units span two or more bays, large incompatibility displacements also develop close to the intermediate column, requiring additional retrofit measures (Büker et al. 2021).

unreliable part

C

T

R^*

(a) Negative moment failure

unreliable part

C

T

R^*

(b) Positive moment failure due to transverse soffit cracking

Figure 4: Alternative load paths for different failure mechanisms

The strongback retrofit's primary purpose is the assurance of life safety. It is crucial to acknowledge that, in common with other retrofit techniques for hollow-core floors, the strongback retrofit does not preclude floor damage (Büker et al. 2021). This fact should be communicated clearly to building owners. Notwithstanding, this retrofit is considered to provide a robust alternative load path, resulting in superior performance compared to many other hollow-core floor retrofits. Besides its overall versatility in addressing the potential failure mechanisms, the strongback solution offers many additional benefits, including:

1. Effectiveness at keeping the floor vertically at the same level as the support. This contrasts with most hollow-core floor retrofits that only engage once the floor unit sustains a considerable drop. The drop is likely to cause delamination of the topping concrete and potentially has detrimental effects on the diaphragm capacity and also compromises the functionality of the floor.

2. Improvements to the overall floor integrity through the presence of post-installed anchors. During the validation test, it was observed that web-cracking consistently terminated before the anchor rows. This positive effect can be attributed to the clamping action generated by tightening the anchors (see Section 3.5 for further elaboration).

3. Design of the strongbacks follows the principles of New Zealand Standards for the design of steel and concrete structures, meaning it should be straightforward to demonstrate compliance with the New Zealand building code,

4. Entire installation of the strongback retrofit can be conducted from the underside of the floor and thereby limits the impact on building operations, and

5. Ease of post-earthquake damage inspection due to the clearance between the seating angle and floor underside provided by the strongback steel beams. Critical damage, in contrast, may be missed when a seating angle installed hard up against the soffit hides this part of the floor.

While the strongback retrofit can significantly improve the life-safety performance of the floor, the material and installation costs are expected to be relatively high, particularly because the relocation of existing services may be required. Furthermore, the installation of the strongback members can be challenging, and attention should be paid to the member weight and how the parts will be lifted into place.

These disadvantages, particularly the cost aspect, draw into question whether the strongback retrofit is required for every hollow-core unit in a building. As discussed in Section 1, seating angles cannot sufficiently address PMF, but the strongback retrofit can do so (see Figure 4b) and has been recommended for this purpose by New Zealand's engineering societies (SESOC et al. 2021). To date, the strongback retrofit is the only experimentally validated retrofit that can dependably address PMF in hollow-core floors. Therefore, the strongback retrofit should at least be installed under hollow-core units for which the PMF drift capacity does not reach the %NBS target.

For cases where the PMF capacity reaches or exceeds the %NBS target, alternative retrofit solutions may be acceptable. For instance, hollow-core units that are only vulnerable to LOS can still sufficiently be retrofitted with just a supplemental seating retrofit (Büker et al. 2022a). Furthermore, hollow-core units that are prone to LOS and NMF (but not prone to PMF) can be retrofitted, for example, with a combination of a supplemental seating retrofit and floor strengthening with additional rebars post-installed in the concrete topping (Parr et al. 2019) as an alternative to the strongback retrofit.

Design and detailing of the strongbacks and their support angle can be challenging for 'alpha units'. Alternative retrofits for alpha units, such as 'catch beams', can be found in the companion paper by Brooke et al. (2022).

In the next section, comprehensive design and detailing recommendations for strongback retrofit are provided.

3 DESIGN AND DETAILING RECOMMENDATIONS

As noted in the previous section, the design of strongbacks follows the principles encompassed by the New Zealand Standards for the design of steel and concrete structures. Further detail regarding a recommended approach to the design and detailing is outlined in this section. This recommended approach incorporates the experience gained during the design, construction, and testing of the super-assembly specimen used for the validation test.

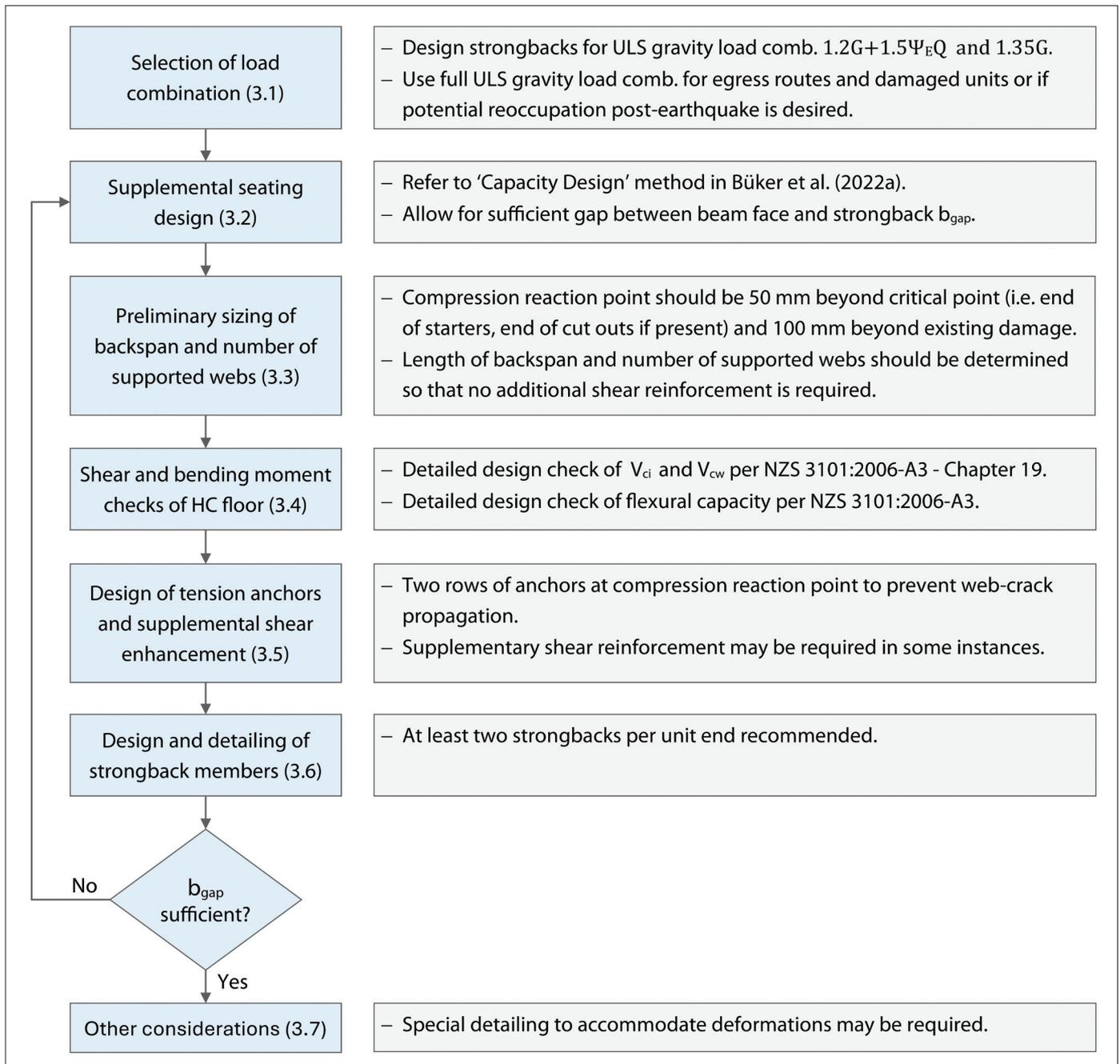The steps encompassed in the design and detailing approach comprise:

1. Selection of load combination.

2. Supplemental seating design.

3. Preliminary sizing of required backspan and number of supported webs.

4. Shear and bending moment design checks of the hollow-core floor.

5. Design of tension anchors and supplemental shear enhancement.

6. Design and detailing of the strongback members.

7. Other considerations.

A flowchart visualising the design procedure with comments on the key considerations for individual design steps is shown in Figure 5. More detailed elaborations of each design step can be found in the following sub-sections.

A site inspection should be conducted prior to designing the strongback retrofit. Aspects to look out for during the inspection are highlighted throughout this section. The inspection is not included as a separate design stage as it is envisaged it may well have been completed as part of an earlier assessment of the floor.

Selection of load combination (3.1)	− Design strongbacks for ULS gravity load comb. $1.2G+1.5\Psi_E Q$ and $1.35G$. − Use full ULS gravity load comb. for egress routes and damaged units or if potential reoccupation post-earthquake is desired.
Supplemental seating design (3.2)	− Refer to 'Capacity Design' method in Büker et al. (2022a). − Allow for sufficient gap between beam face and strongback b_{gap}.
Preliminary sizing of backspan and number of supported webs (3.3)	− Compression reaction point should be 50 mm beyond critical point (i.e. end of starters, end of cut outs if present) and 100 mm beyond existing damage. − Length of backspan and number of supported webs should be determined so that no additional shear reinforcement is required.
Shear and bending moment checks of HC floor (3.4)	− Detailed design check of V_{ci} and V_{cw} per NZS 3101:2006-A3 - Chapter 19. − Detailed design check of flexural capacity per NZS 3101:2006-A3.
Design of tension anchors and supplemental shear enhancement (3.5)	− Two rows of anchors at compression reaction point to prevent web-crack propagation. − Supplementary shear reinforcement may be required in some instances.
Design and detailing of strongback members (3.6)	− At least two strongbacks per unit end recommended.

No ← b_{gap} sufficient?

Yes ↓

Other considerations (3.7)	− Special detailing to accommodate deformations may be required.

Figure 5: Flowchart outlining the recommended design and detailing procedure for the strongback retrofit. Numbers in brackets refer to section numbers that follow.

3.1   SELECTION OF LOAD COMBINATION

The first step of the design process is to select an appropriate load combination. The design of the strongback retrofit needs to meet the ultimate limit state (ULS) requirements defined in the Structural Design Action Standard NZS1170.0:2002-A5 (SNZ 2011). Based on this standard, the retrofit needs to be able to sustain two gravity load combinations:

$$E_d = 1.35G \qquad (1)$$

$$E_d = 1.2G + 1.5Q \qquad (2)$$

where G is the permanent action ('dead' load) and Q is the imposed action ('live' load).

In a companion paper, Brooke et al. (2022) argue that it is unlikely that the full imposed action, Q, acts on the damaged flooring units during or after the earthquake (provided access is restricted). Therefore, the following load combination as defined by Brooke et al. may be used instead of Equation (2) for the design of floor retrofits:

$$E_d = 1.2G + 1.5\psi_E Q \qquad (3)$$

where ψ_E is the combination factor for earthquake action and all other parameters have been defined previously.

Before designing the strongback retrofit with Equation (3), the following factors should be considered:

- Strongback retrofits for hollow-core units that are part of an egress route should be designed for the full ULS gravity load (Equations (1) and (2)) because the gravity demands on such units may likely see high live loads during the evacuation in an earthquake event.

- It is crucial to recognise that the building will not be able to be re-occupied following an earthquake if the hollow-core units are supported by strongbacks designed with Equation (3). This constraint needs to be communicated clearly to building owners and tenants.

- In cases where a hollow-core unit shows existing damage that may impair the gravity load-carrying capacity (i.e. diagonal web-cracking), it is advisable to design the strongback retrofit (or any other retrofit) to support the full ULS gravity load (Equations (1) and (2)) for continued occupancy.

Refer to the companion paper by Brooke et al. (2022) for further discussion in regards to the selection of appropriate load combinations for the design of floor retrofits.

Once the appropriate design load combination is identified, the support reaction, R^*, can be determined.

3.2   SUPPLEMENTAL SEATING DESIGN

After choosing an appropriate load combination, the supplemental seating can be designed. It is recommended to follow the capacity-design approach[1] described in the seating angle design recommendations (Büker et al. 2022a).

The capacity design approach is generally applicable when used in conjunction with the strongback retrofit, but the seating length calculation requires special consideration in this case.

With the strongback being the point of contact to the seating angle, the additionally provided retrofitted seating length, $\delta_{s,ret}$, should be measured relative to the end of the strongback steel member as indicated in Figure 6. The seating movement demand, δ_s^*, may then be calculated as the sum of movement due to beam elongation and relative rotation at the height of the seating angle top flange based on the recommendations by Büker et al. (2022a). The demands on the seating must not exceed the available retrofitted seating:
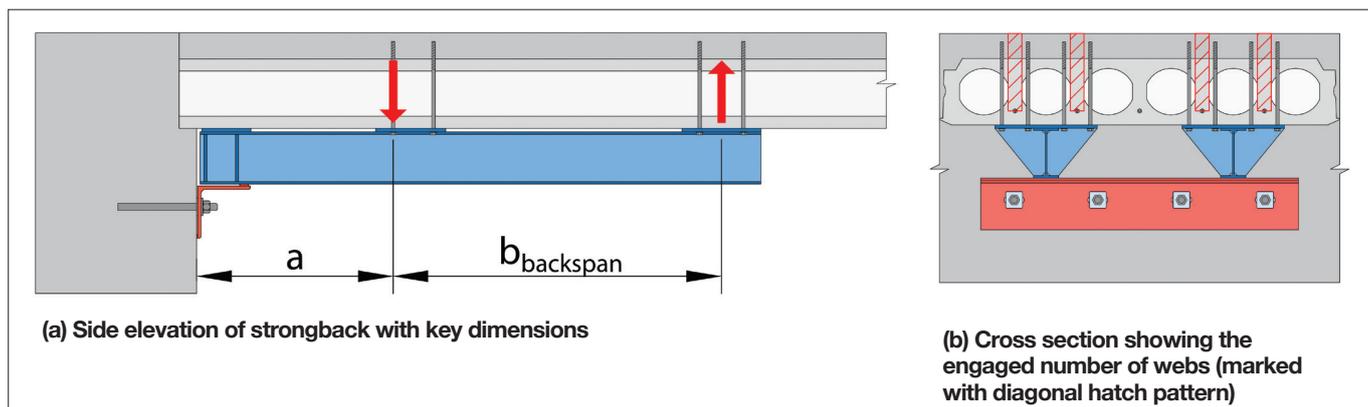
$$\delta_s^* \leq \delta_{s,ret} \qquad (4)$$

As indicated in Figure 6, a gap, b_{gap}, between the support structure and strongback retrofit is required to allow for movements due to relative rotations between the floor and the support beam. If the size of the gap is insufficient, the
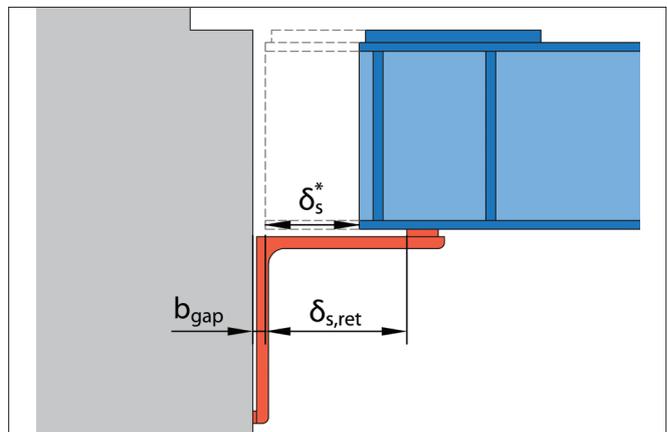
Figure 6: Seating conditions when using the seating angle with the strongback retrofit

strongback may come into contact with the support beam, resulting in relative movements between the strongback and the floor. These relative movements can have the potential to shear off the heads of the post-installed vertical strongback anchors. This critical damage is of particular concern where large relative rotation demands between

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[1]   If using the capacity-design approach is impractical, the dropped-angle design approach as defined in the companion paper by Büker et al. (2022a) may be used as an alternative.

(a) Side elevation of strongback with key dimensions

(b) Cross section showing the engaged number of webs (marked with diagonal hatch pattern)

Figure 7: Key dimensions and design parameters for the strongback retrofit

the floor and support without the displacements due to elongation are possible. Without knowing the exact depth of the strongback retrofit, the gap size cannot be determined at this stage.

Therefore, a 15 mm gap may be assumed, which will need to be reviewed once the strongback depth is confirmed (refer to Section 3.6). To ensure the gap is implemented as specified, a block of low-density polystyrene or similar compressible material can be placed between the strongback and the face of the support beam during the installation.

If desirable for practical reasons, the total depth of the retrofit installation could be minimised by installing the seating angle upside down.

3.3 PRELIMINARY SIZING OF REQUIRED BACKSPAN AND NUMBER OF WEBS

A key design consideration for the strongback retrofit is to determine a configuration of strongbacks that ensures sufficient shear capacity of the hollow-core webs within the backspan, $b_{backspan}$, which is the region between the tension reaction, T, and compression reactions, C, as indicated in Figure 7a. Along the backspan, the shear demand on the webs is increased due to the transfer of forces out of the hollow-core unit and into the strongback. In addition, the number of engaged hollow-core webs, n, that can be relied upon for the shear capacity is reduced. As indicated in Figure 7b, only the webs supported by the strongback at the compression point, C, can be considered for the shear capacity. The figure also demonstrates that it is possible to support several webs with one strongback by adding outriggers at the compression point and tension point.

The distance between the beam face and the compression point (designated as distance "a"), depends on a number of factors, such as the floor dimensions,

critical failure modes and potential existing damage. The following guidance is provided in the selection of distance a:

$$a \geq \max \begin{cases} 1.5 \ h_{floor} \\ l_{starters} + 50 \text{ mm} & \text{when NMF is possible,} \\ l_{cutout} + 50 \text{ mm} & \text{when the unit has cut-outs,} \\ l_{damage} + 100 \text{ mm} & \text{when there is pre-existing damage.} \end{cases} \quad (5)$$

where h_{floor} is the depth of the floor including the topping, $l_{starters}$ is the extension of the starter bars into the topping beyond the beam face, l_{cutout} is the length beyond the beam face of any cut-outs at the unit end (e.g. to accommodate a column) and l_{damage} is the distance from the beam face to the furthest point of existing external and internal damage in the unit. Note that all distances a and l are measured from the beam face.

Following the determination of distance a, the required backspan, $b_{backspan}$, and the number of webs engaged by the strongbacks, n, can be determined. Governing factors for the determination of these values are typically the web-shear capacity and the flexural-shear capacity as defined in Section 19 of NZS3101:2006-A3 (SNZ 2017). Further guidance on the determination of these shear capacities is provided in the subsequent Section 3.4.

An estimate for the required backspan length, $b_{backspan}$, based on distance a and the number of engaged webs, n, can be obtained from the preliminary design charts shown in Figure 8. These graphs were generated based on the shears and moments that can be withstood by the hollow-core floor without the installation of supplemental shear reinforcement. The applicability of these charts is limited to Equation (1) and the post-earthquake retrofit load combination (Equation (3)) with uniformly distributed live loads of 3 kPa for office use in accordance with NZS1170.1:2002-A2 (SNZ 2009). While these preliminary design charts may be useful for the initial sizing, the charts should not be used as a substitute for the detailed design checks described in the subsequent sections.
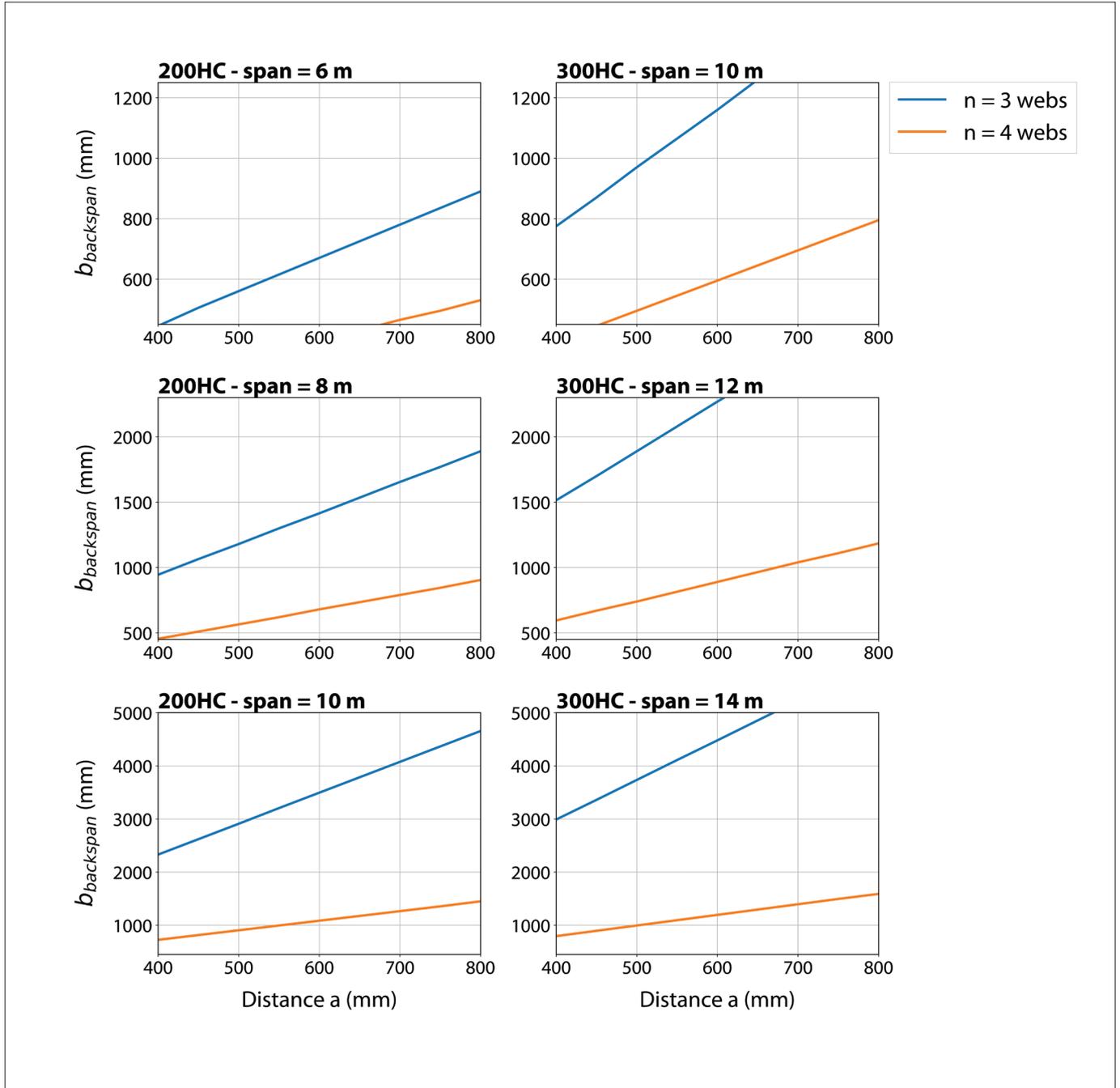
Figure 8: Preliminary design charts to determine the backspan length, $b_{backspan}$, and the number of engaged webs, n assuming no additional shear reinforcement (Not to be used for detailed design)

As a conservative assumption, distance a is defined as the distance from the beam face to the location of the compression reaction, C. This assumption is based on the possibility of the front end of the strongback coming into contact with the supplemental seating retrofit. This scenario has been observed at large lateral drifts in excess of 3% in the validation test and is further discussed in Section 3.6 and shown in Figure 14. A less conservative assumption for distance a may be warranted if contact between the front end of the strongback and the angle can be avoided, i.e. a sufficiently deep strip of steel is

installed at the tip of the seating angle or supplemental seating alternative. If it can be demonstrated that this contact is prevented, the distance between the support reaction and the compression reaction point may be refined to:

$$a_{refined} = a - b_{gap} - \delta_{s,ret} + \delta_s^* \qquad (6)$$

where a is calculated based on Equation (4) and all other parameters have been defined previously.

The value of $a_{refined}$ can replace distance a and used with the preliminary design charts.

3.4 SHEAR AND BENDING MOMENT DESIGN CHECKS OF THE HOLLOW-CORE FLOOR

After the preliminary sizing, detailed design checks of the shear and moment demand in the floor unit should be conducted. These design checks require careful consideration because (1) a reduced number of webs is being relied on to resist the shear demands and (2) the shear demands are increased compared to a simply supported undamaged floor because the compression reaction C is the sum of the support reaction, R^*, and the tension reaction, T.

Earthquake damage is expected to concentrate between the support and the compression point (i.e. within span a). Consequently, the bond between prestressing strands and concrete along length a may be heavily impaired and, hence, unreliable. This implies that the transmission of the prestress now starts at the compression point (Figure 9b). The prestress transmission can be assumed as linear over the transfer length, L_t, as defined in NZS3101:2006-A3 (SNZ 2017). Taking this assumption into account, the shear capacity for both web-shear cracking, V_{cw}, and flexural-shear cracking, V_{ci}, as specified in Section 19 of NZS3101:2006-A3 (SNZ 2017), can be calculated. Only the selected number of engaged webs n should be considered when calculating the shear capacities (Figure 7b). An on-site inspection is required to determine the number of strands per unit and **specifically ensure that the webs engaged by the strongback retrofit contain strands.**

The concrete strength of existing hollow-core units has been found to be significantly greater than the specified 28-day strength, which is commonly specified as 45 MPa (Fenwick et al. 2010). Using this as a basis, it is appropriate to use the probable concrete strength, which can be determined by multiplying the specified 28-day concrete strength by 1.4 based on Section C5.4.2.2 in the Assessment Guidelines C5 (MBIE et al. 2018). For the determination of the shear capacity, NZS3101:2006-A3 (SNZ 2017) limits the concrete strength value to 50 MPa. Other standards (i.e. ACI318 (2019)) have higher concrete strength limits for the calculation of shear capacities and should be referred to when intending to utilise the full probable concrete strength for the shear design. If this approach is adopted, it should be noted that this would be considered an "Alternative Solution" because the other standards are not cited in the NZ Building Code B1/VM1 (MBIE 2019).

Furthermore, the flexural capacity according to NZS3101:2006-A3 (SNZ 2017) needs to be checked against the flexural demands along the backspan, $b_{backspan}$, and the rest of the floor. When calculating the flexural capacity of the floor, it is important to recognise the relocated development length location of the strands as outlined above.

The shear and bending moment distributions should be plotted along the length of backspan, $b_{backspan}$, as shown in the conceptual example in Figure 9. If it is found that the shear or bending demands exceed the capacity along the length of the floor, the assumed backspan, $b_{backspan}$, and number of engaged webs, n, can be increased or, alternatively, the shear capacity can be enhanced with supplemental shear reinforcement as discussed in the subsequent section.
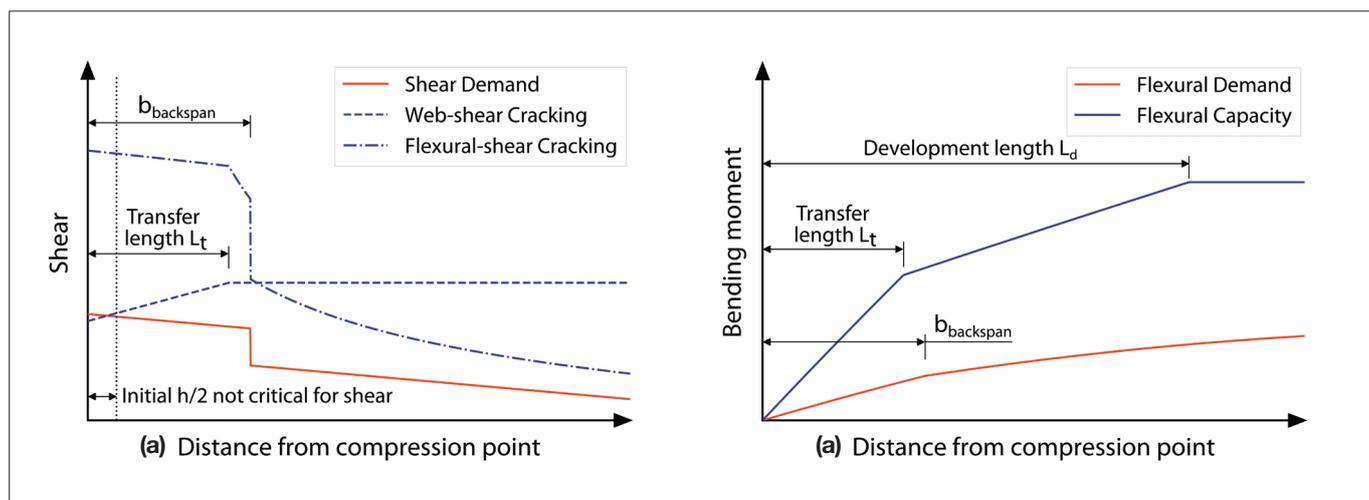

Figure 9: Example of shear and bending moment distribution plotted from the compression point towards mid-span of the floor

3.5 DESIGN OF TENSION ANCHORS AND SUPPLEMENTAL SHEAR ENHANCEMENT

Vertical anchors between the strongbacks and the hollow-core floor are an essential part of the strongback retrofit technique. The anchors have three critical functions, namely:
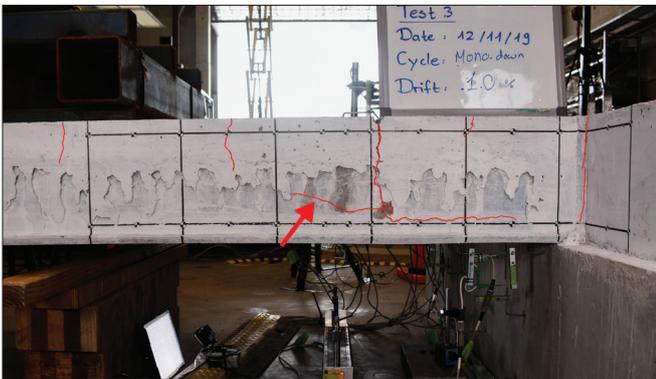
- Transfer of the tension force, T, from the far end of the strongback into the floor,

- Supplemental shear reinforcement, and

- Clamping action at the compression point.

The need for a tension load path at the far end of the strongback and the potential need for supplemental shear reinforcement is self-evident. The importance of the clamping action provided at the compression point is not as intuitively crucial; however, experimental evidence from previous studies highlights the importance of such

Figure 10: Side view of a hollow-core floor with a negative moment crack (marked in red) that propagates through the webs and branches horizontally towards and away from the support. *(Photo taken at -1.0% inter-storey drift) (Büker 2022)*

a clamping mechanism to be present for the strongback retrofit. A number of studies have investigated the NMF mechanisms in hollow-core floors (Büker (In Preparation); Liew 2004; Parr et al. 2019; Woods 2008). As illustrated in Figure 10, the negative moment crack would typically propagate vertically through the webs, then branch horizontally in both directions – towards the support and towards the mid-span of the floor. Such horizontal web cracks would critically impair the shear resistance along the backspan, potentially resulting in a brittle shear failure.

Experimental observations from the validation test indicate that the clamping forces generated by the screw-anchors that were installed at the compression point effectively prevented the propagation of internal floor damage (i.e. web-cracking) beyond this point.

In the validation test, one of the hollow-core units sustained structurally non-critical damage to the unreinforced outer web when the specimen was loaded to 4% inter-storey drift. During subsequent loading to 5% inter-storey drift, parts of this outer web collapsed and thereby exposed the interior of the cell, which is illustrated in Figure 11. To the right of the compression point, C, a large web-crack caused by NMF can be seen branching from the top of the floor towards the support. No web-cracking was observed to propagate in the opposite direction beyond the vertical anchors, which provided the clamping action. This observation was not limited to the hollow-core unit illustrated in Figure 11 but was a consistent finding for all hollow-core units in the validation test.

Figure 11: Strongback retrofit carrying the load of the floor after a negative moment failure has occurred.
(Photo taken at 5% inter-storey drift)
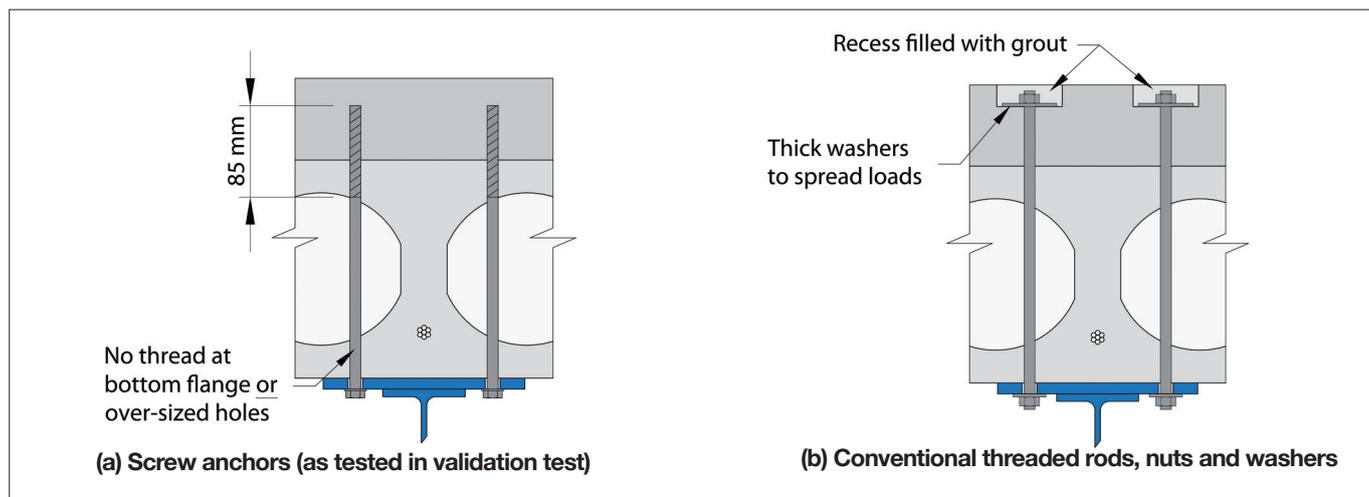
Figure 12: Example of different anchor solutions

To provide the clamping action, two rows of screw anchors with a diameter of 10 mm should be installed along each engaged web at the compression point (i.e. four anchors per engaged web). These anchors should be snug tight so that some degree of vertical pretension on the webs is provided. For the validation test, the screw-anchor product *fischer UltraCut FBSII-10x260* was used. If a different anchor solution or diameter is selected, it is recommended to determine the required number of anchors at the compression point based on the condition that the modified anchor configuration should provide an equivalent or greater stiffness per engaged web compared to four of the 10 mm screw-anchors used in the validation test. These four screw-anchors provided a stiffness of 0.36 kN/m over their unbonded length. No less than one row of anchors (i.e. one anchor on each side of an engaged web) should be installed at the compression.

If the shear capacity of the precast unit within the backspan is insufficient, supplemental shear reinforcement can be provided to ensure sufficient shear capacity. This can be achieved by installing additional anchor rows along the engaged webs. The arrangement of the supplemental anchors should satisfy shear reinforcement spacing provisions in NZS3101:2006-A3 (SNZ 2017).

In the validation test, five rows of anchors per engaged web were installed starting from the compression over a portion of the strongback's backspan (see Figure 11). The first two anchor rows had the main purpose of providing the clamping action that restrains cracking of the webs and contributes to the shear capacity. An additional three anchor rows were found to be required as supplemental shear reinforcement to enhance the

shear capacity along this part of the backspan. While the anchors that provide the clamping action are always required, the number of additional supplemental shear anchors can be reduced or completely omitted by increasing the backspan length, $b_{backspan}$, or increasing the number of engaged webs, n.

If the strongback retrofit is intended to address web-splitting due to incompatibility displacement in alpha units, the anchor configuration should be similar to what was tested in the validation test (Figure 3) unless it can be shown that a detailing variation can successfully address this failure mechanism.

All post-installed anchors should be designed in accordance with EN 1992-4 (CEN 2018) (superseded EOTA TR045, which is referenced in NZS3101:2006-A3 – Cl. 17.5.5 (SNZ 2017)). The selected screw anchor product for the validation test had a partial thread that was embedded into the topping concrete and top flange of the unit, as shown in Figure 12a. The screw-anchors were located close to the webs to achieve a sufficient embedment in the curved portion of the cell and allow for a direct load transfer of the clamping force from the anchors to the webs.

Using screw-anchors has the benefit that the installation of the strongback retrofit only requires access to the floor underside. Nevertheless, some additional design and detailing considerations should be taken into account when using screw anchors. Firstly, it is important that screw-anchors only anchor into the top part of the floor in view of the fact that anchoring into the bottom flange may promote web-cracking. For screw-anchor solutions with a full-length thread, oversizing the holes in the bottom flange of the hollow-core unit can prevent this detrimental effect. Secondly, the validation test showed

that screw-anchors with a larger diameter (i.e. 10 mm or greater) generally provide more redundancy when crossed by cracks such as longitudinal splits. When short screw-anchors with a diameter of 6 mm were tested, it was found that longitudinal splits in the topping and top flanges of the hollow-core units significantly impaired their load capacity. As a result, some of the 6 mm diameter screw-anchors loosened to the extent that a few of the anchors fell out of the floor. Therefore, it is recommended to use larger diameter screw-anchors, which may lead to a low demand-capacity ratio.

As an alternative to using screw anchors, it may be more desirable to drill through the entire floor depth and use conventional threaded bars, nuts and washers as anchor solution. At the top, the washers and nuts can be recessed into the topping and the remaining pocket filled with high-strength grout, as depicted in the detailing example in Figure 12b. This anchor solution was conceptually tested by Jensen (2006). The experimental work by Jensen also showed that the holes should be drilled from the underside because drilling from the top can result in large concrete cone breakouts in the bottom flange of the hollow-core unit. Sizing of these conventional products should be conducted following recognised procedures.

While there may be other suitable anchor solution alternatives, the use of expanding anchors is discouraged as this type of anchor may promote longitudinal splitting or other cracking to form through the anchor lines due to the induced transverse tensile stresses from the anchor expansion.

Irrespective of the method used to post-install anchors and transverse reinforcement, it is critical that care be taken to avoid damage to the prestressing strands in the hollow-core units.

3.6 DESIGN AND DETAILING OF THE STRONGBACK MEMBERS

The strongback steel beams can be designed to resist the imposed demands, allowing for flexibility in the detailing. I-sections are generally preferred over hollow sections for the steel beams because I-sections have many benefits in terms of constructability. As shown in Figure 13, the I-beams in the validation test were set down by 10 mm from the soffit of the hollow-core unit with only three critical points of contact with the floor unit:

- A plate welded onto the top flange of the I-beam close to the support beam. If LOS occurs, a direct vertical load transfer from the unseated floor through the strongback into the seating angle can be established.

- A second point of contact is required where the compression force is designed to land. This contact point may be detailed with a flat plate that is fixed to the hollow-core unit by post-installed anchors. Ideally, this plate should not be rigidly connected to the I-beam so that relative sliding movements between the I-beam and the floor can be accommodated. In some cases, the plate at the compression point may need to be welded to the I-beam, for example, when a strongback is designed to support multiple webs. In order to prevent the anchor heads from getting sheared off due to the relative laminar shear sliding movements, the holes for the anchors need to have sufficient tolerances.

- The third point of contact is where the tension force needs to be transmitted between the floor and strongback. At the tension point, the holes for the anchors are also recommended to have sufficient tolerances for the aforementioned reasons.
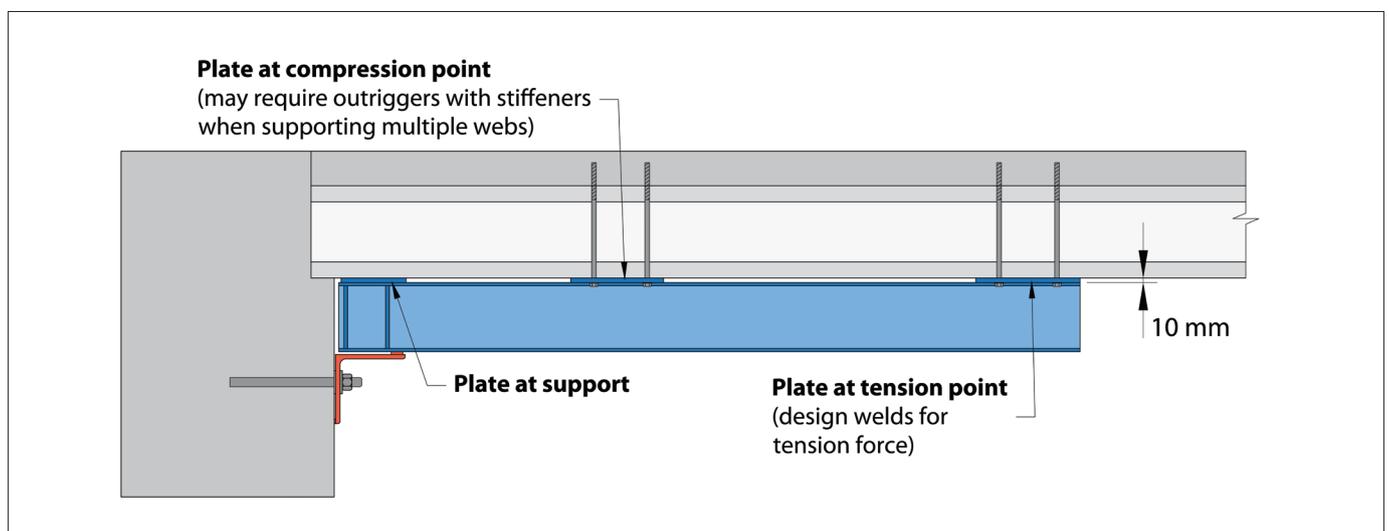

Figure 13: Typical strongback side elevation

Where a single strongback is intended to engage multiple webs, the flat plate at the compression point needs to be extended to the sides with vertical stiffeners that are welded to the flat plate and the I-beam. This arrangement forms wide outriggers that provide sufficient stiffness to support several webs (conceptually demonstrated in Figure 7b). When multiple webs are engaged by one strongback, the tension anchors at the far end of the strongback should be equally distributed to the engaged webs.

Although it may be possible to use a single strongback member per unit end and, in fact, has successfully been tested in the validation test, a minimum of two strongbacks per unit end is recommended to ensure torsional stability and to provide a degree of redundancy. This restriction is imposed due to the possibility of a longitudinal split forming directly above a single strongback. The longitudinal split could critically decrease the load-carrying mechanism of the strongback and may result in the collapse of the unit. Two or more strongbacks, in contrast, will provide the necessary redundancy.

The general design and detailing of the strongback members can be conducted utilising conventional steel design procedures and standards. Following these procedures, it can be found that the required section size of the steel beam is typically small. Consideration should also be given to the stiffness of the strongback elements in relation to the expected floor deflections. While no prescriptive criteria are suggested here, it is recommended to generally keep the deflections of the strongback low so that the diaphragm remains on the same level. The strongback and floor deflections should be assessed with the assumption that the unreliable part of the floor (refer to Figure 4) becomes completely ineffective. When sizing the steel members, it is important to ensure that the steel beam has sufficient capacity to withstand lateral-torsional buckling, particularly where distance a is of significant length. Furthermore, the potential of local buckling of the web of the strongback must be checked at (1) the compression point and, more importantly, (2) where the strongback is supported by the seating angle. It is crucial to recognise that the demand on the strongback webs at their front end is not limited to the support reaction, R^*. As discussed, in the seating angle design recommendations (Büker et al. 2022a), the compression reaction at the tip of the angle can grow as large as the force required to reach the overstrength moment of the seating angle. This higher force only needs to be considered for the local buckling check of the strongback webs at the steel strip of the

supplemental seating retrofit. Note that the location of the force relative to the strongback shifts with increasing beam elongation demands.

During the validation test, it was observed that contact between the front tip of the strongback and the seating angle could occur at large inter-storey drifts (Figure 14). Due to the advanced floor damage at large inter-storey drifts, the floor-to-support connection will have softened significantly at this stage. On this basis, it can be assumed that the magnitude of this new reaction at the front end of the strongback is unlikely to significantly exceed the value of the support reaction, R^*. To ensure that the strongback remains undamaged in this critical location, it may be advisable to add stiffeners as a precautionary measure, as had been done for the tested strongbacks (see Figure 14). Alternatively, a sufficiently thick steel strip at the tip of the angle could be installed (refer to Section 3.3 for further discussion).

Figure 14: Front end of strongback in contact with the seating angle (first observed at 3% inter-storey drift).

Once the steel member dimensions are confirmed, the assumed required width of the gap between the support beam and strongback, b_{gap}, should be revisited. Knowing the depth of the strongback member, the required gap can be calculated as follows:

$$b_{gap} = h_{stb}\, \theta_{MCE} + 5 \text{ mm} \geq 15 \text{ mm} \qquad (7)$$

where h_{stb} is the depth of the strongback and θ_{MCE} is the peak MCE inter-storey drift demand, which is defined as the peak ULS inter-storey drift demand multiplied by $1.5/S_p$ based on the provisions in NZS3101:2006-A3 (SNZ 2017). If the calculated required gap exceeds the previously assumed gap, the seating angle design needs to be revised with the updated larger gap size.

The above recommendations on strongback detailing are based on what successfully worked in the validation test. The authors recognise that detailing deviations may be required based on floor and site conditions, material choice, etc.
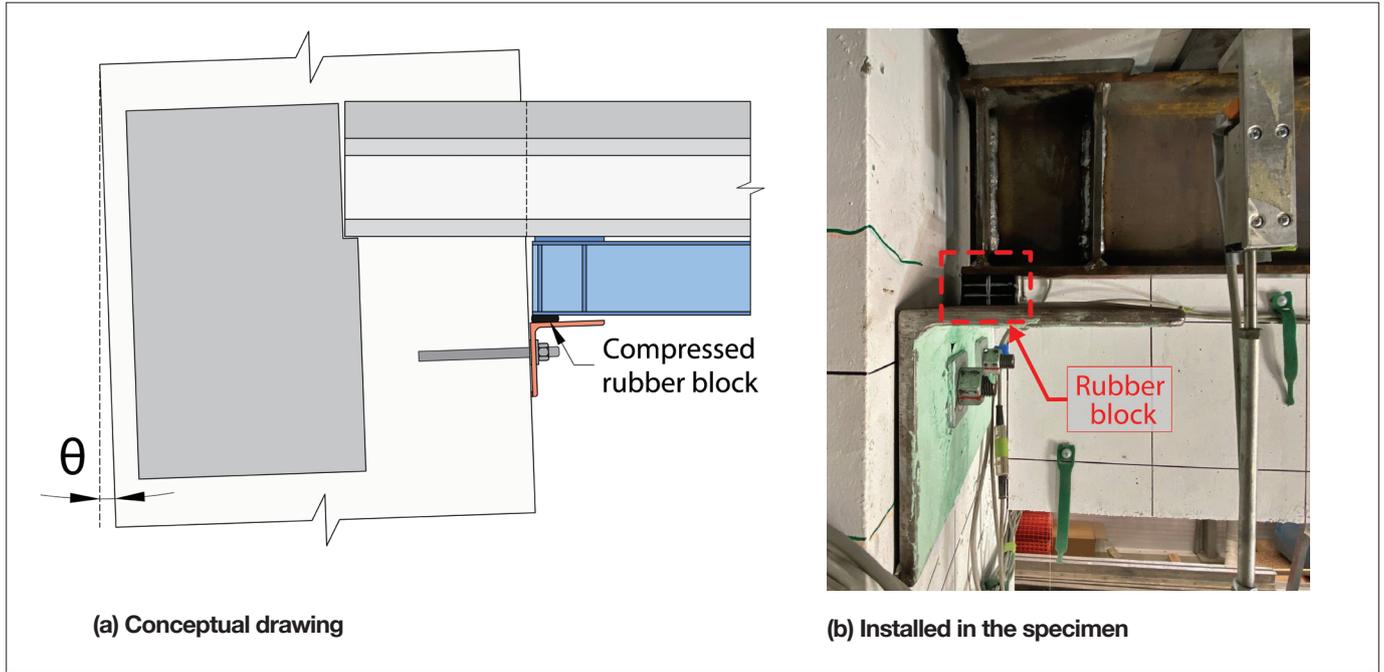
Figure 15: Rubber block detail used to accommodate relative vertical movements at the column face

3.7 OTHER CONSIDERATIONS

The preceding sections have dealt with the design approach required when strongback retrofits are considered for 'typical' hollow-core units. This section describes additional detailing considerations which may be required for specific floor support arrangements.

Detailing to accommodate displacement compatibility may require particular attention in the vicinity of the columns. The specimen for the validation test, for instance, required some of the floor units to have cut-outs allowing the columns to protrude into the floor. As a result, a number of strongbacks needed to be seated on a seating bracket attached to the column face (Büker et al. 2021). This detail, however, raises the issue that, due to the distance between the support beam and the column face, large relative vertical movements at the column face arise from lateral drifts (Figure 15a). These vertical movements have the potential to cause undesired critical damage to the angle anchor bolts and the floor unit. By adding a block of unreinforced natural rubber (Figure 15b), large forces could be avoided while still ensuring that the strongback is sufficiently supported.

These rubber blocks were designed with the assumption that the vertical deformation of the block at 2.5% inter-storey drift generates a force equal to the maximum ULS gravity load in accordance with Section 3.1. Simple compression tests were conducted to find the appropriate dimensions and suitable shore hardness of the rubber. The rubber block was glued to the

underside of the strongback so that it can slide on the seating angle. The seating angle can be designed with the 'dropped angle' approach, which is described in Büker et al. (2022a). Note that while the rubber block is useful in this instance to prevent undesired damage, it is not recommended to use rubber blocks as a general alternative to the 'capacity design' solution for the strongback retrofit design.

Finally, when establishing a retrofit strategy for a given floor plate, a range of retrofit solutions may be used for different precast floor units. When selecting suitable retrofits, it is important to ensure compatibility between the individual retrofits. One of the main advantages of the strongback retrofit is the ability to keep the floor vertically at the same level as the support, while other retrofits may only engage once the floor drops. For this reason, the strongback retrofit should preferably only be used in conjunction with other retrofits that do not allow the units to drop.

4 CONCLUSION

The strongback retrofit addresses the majority of failure modes in hollow-core floors and, most notably, is a suitable retrofit for both negative moment failure and positive moment failure. Experimental testing has successfully validated the effectiveness of this new retrofit technique. The validation test highlighted the benefits of the strongback retrofits, such as the load path integrity provided by post-installed shear anchors and the ability

to keep the diaphragm on the same level. Due to the relatively high material and installation costs, the use of the strongback retrofit may be limited to the critical hollow-core units (i.e. where PMF is a concern).

The recommended design steps described in this paper offer a straightforward design procedure. The most important aspects to consider when designing the strongback retrofit are summarised as follows:

1. An appropriate load combination should be selected based on the performance objective.

2. The seating angle should be designed following the 'capacity design' procedure described by Büker et al. (2022a) with modifications for the seating length design (see Section 3.2).

3. The length of the strongbacks highly depends on the number of hollow-core floor webs engaged and the shear capacity of these webs. Preliminary design charts are provided in this paper, but a detailed design check of the floor capacities is still required as an essential part of the strongback design.

4. Two rows of screw anchors with a diameter of 10 mm (or anchors with equivalent stiffness to this configuration) should be installed at the compression point to prevent web-crack propagation. Supplemental shear reinforcement must be provided if the shear capacity of the hollow-core webs is insufficient.

5. Careful detailing of the steel components is required to ensure stable transfer of loads between the unit, strongback beam, and support angle.

5   ACKNOWLEDGEMENT

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BUILDING PERFORMANCE CONSULTATION DOCUMENT

Consultation document Building Code *update 2022 Structural stability of hollow-core floors,* amending Verification method B1/VM1 (2 May 2022).

The building regulator has commenced with the 2022 building code update consultation. This includes proposing changes to the compliance pathway for hollow-core floors. The change that is being consulted on proposes the removal of a 'deemed to comply' solution for the support of hollow-core floors in new building designs. It is proposed that the design of hollow-core floor systems will still be possible via an 'alternative' solution pathway. Consultation will run until 1 July 2022.